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PERFORMANCE PREDICTION AND USES OF PV BAND DRAINS UNDER THE EMBANKMENTS ON SOFT MARINE CLAYS OF BANGKOK

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ABSTRACT

The use of prefabricated vertical (PV) drains for the improvement of soft clays is a widely adopted technique today. Vertical drains are used to accelerate the consolidation under preloading by taking the advantage of higher rate of horizontal flow of water through the soil. The present paper is aimed at the prediction of the performances shown by the 'Alidrain', used in the soft marine clays of Bangkok, in the field under embankment loading and in the laboratory in a in-house designed large scale consolidometer, by using a Finite Element model of the transient, axi-symmetric flow problem considering equal strain. This model also considers the disturbances caused by the driven mandrels to the surrounding subsoil (smear effect) and the effect of the sizes of mandrel used for installation. The data collected from field and laboratory has been compared with the results from this model. The model demonstrated good prediction of both the field and laboratory performances with proper choice of the C_{field}/C_{lab} ratio, the smear zone diameter (d_s), the k_h/k_h' ratio for the smear zone and the other input soil parameters found from different laboratory experiments. For the prediction of consolidation settlement in the field, a proposed value of k_h/k_h' ratio around 10 with the combination of $d_s=2d_m$ has been found to give the best results.

KEYWORDS

PV Drains, Consolidation (Coefficients), Permeability (coefficients), Smear, Mandrel, Marine clay, FEM, Settlement, Investigations.

INTRODUCTION

Geotechnical engineering is essentially concerned with the understanding of the soil behaviour and utilisation of soil deposit that covers about one-third of the land mass on earth. If one goes through the complicated process of the formation of the earth's crust that took place over millions of years, it is not difficult to perceive the vast diversity of this huge mass in geology and composition. The task that is more difficult is to make it behave the way we want and this is the point where a geotechnical engineer has to face the greatest testimony of his knowledge and judgement in coping with the uncertainties in the soil behaviour.

One of the major problem which have been outwitting the soil engineering people over years is settlement. Settlement reveals the deformation characteristics of the soil material over time. Basically, it can be taken as a physical response of the soil to the externally applied loads on it. Even without going into the detailed analysis of the structures, formation and depositional environment of the soil particles, one can therefore, rather intuitively, feel that stiffer the soil, lesser the amount of voids in it and fewer the involvement of water, higher the ability of the soil deposit to bear loads and thus, more strong it is.

With the fast growth of human population and rapid industrialisation all over the world, good sites are fast getting extinct from the face of the earth. As a matter of fact, soil investigations today are more concerned with selecting a site where soil conditions are not that adverse. Therefore, ground improvement can not be avoided in any way. The stress has to be given on the right choice of technics, applicability of the method as per the demand of the project and existing ground conditions, and finally economy and reliability of the whole affair for the sake of optimum utilisation of land area.

Out of various ground improvement technics, this study will only concern with the suitability and performance prediction of a definite type of pre-fabricated vertical (PV) drain in soft marine clays of Bangkok city. The vertical drains are used to accelerate the rate of consolidation by preloading by taking the advantage of reduction of the drainage path and higher rate of consolidation drainage in radial direction. If we take close look at the subsoil profile at Bangkok, we will observe that the top 2m consists of soft to medium, brown, weathered clay followed by a layer of soft, compressible clay upto the depth of 6m and a silty clay layer upto 8m underneath. The PV drain seem to be very much suitable for this kind of subsoil profile. The present study emphasised on the comparison of the performance of 'Alidrain' (a type of PV drain manufactured in Canada) in the field and in the laboratory and the results has also been compared with the output of a finite element numerical model of the transient, axisymmetric consolidation flow problem considering equal strain and smear effect. This kind of studies are made to validate the utility of laboratory experiments as well as the use of numerical modelling for the prediction of settlement in a vertical drain system in actual in-situ conditions.

SCOPE

This study required the monitoring of the field performance of the 'Alidrain' under the embankment loading. This embankment is situated in the campus of Asian Institute of Technology (AIT) at Bangkok. Performance of the same 'Alidrain' with small mandrel, inserted in the same soil sample, has also been tested in the laboratory to study the smear effect using large scale consolidometer. The sample for the laboratory testing was collected from the same site and has been reconstituted in the laboratory. Finally, the results obtained from the field and laboratory was compared with the results predicted by the numerical modelling of the vertical drain consolidation problem. The values of the soil index properties, compressibility parameters and smear zone parameters those have been used in the model were obtained from the different laboratory and field experiments.

BACKGROUND

Development of Vertical Drain Theories

The basic theory behind the radial consolidation in the vertical drain system is nothing but the extension of Terzaghi's one-dimensional consolidation theory. Since it is a proven fact today that for clays, in general the coefficient of consolidation in the horizontal direction is much higher than that in the vertical direction, the effectiveness of vertical drains in accelerating the rate of consolidation and strength of soft clays is readily understandable. In the year 1935, a solution was presented by RENDULIC; but this theory did not take into account the deviations from the ideal well conditions.

The problem of how vertical drains will affect the process of consolidation in low-permeable soil, such as clay was probably first solved by KJELLMAN (1948) of Swedish Geotechnical Institute. He based his solution on the "equal vertical strain hypothesis", i.e. on the assumption that horizontal sections remain horizontal throughout the consolidation process. However, the doubtless most well known contribution to the solution of the problem of consolidation by drain wells was presented by BARRON (1948). He studied the two extreme cases of 'free strain' where no arching in the soil is assumed, and 'equal strain' and showed that the average consolidation obtained in this cases are very nearly the same. Thus, even if the free strain analysis would physically agree better with reality than the equal strain analysis, which from the experience of the case records is doubtful, it is not necessary to use the very complicated solution obtained by this method. In his analysis, BARRON (1948) also considered the influence of smear and well-resistance on the consolidation process. Smear is the condition in which the permeability of soil adjacent to walls of vertical drain is reduced because of the disturbance occurring during installation of the drain. In the case of well-resistance, solutions were presented for both free and equal strain while in case of smear only a solution for the case of equal strain was presented. TAKAGI (1957) extended Barron's theory to incorporate a variable rate of loading. Convenient design charts for the effect of smear were developed by RICHART (1959), who also considered the effect of a variable void ratio.

All the solutions mentioned above are based on the assumptions that Darcy's Law is valid. However, it has been shown by many researchers that this may not be true when the hydraulic gradient is in the range of magnitude prevailing during most consolidation process in practice. A solution to this problem was given by HANSBO (1960), but in this equal strain solution smear and well resistance were not considered. However, a simpler solution to the problem of smear and well resistance came from HANSBO (1979), giving results almost identical with those presented by BARRON (1948). This solution incorporates important parameters such as vertical discharge capacity, remoulding effects during installation and filter resistance. Most of these theories are based on one more basic assumption that the soil is homogeneous throughout the depth and consolidation parameters does not change with time. This is not true for most of the soils in practice. The theories developed at a later stage and particularly based on numerical technics has taken care of this problem.

Review of Latest Theories

The recent developments in the field of PV drain mostly came from the works by Hansbo. Barron, developed the following consolidation equation for radial drainage:

$$\partial u/\partial t = C_h\,(\partial^2 u/\partial r^2 + \partial u/r\partial r) \qquad (1)$$

combining this with the Terzaghi's one dimensional theory, we can get the governing equation for the drainage in vertical drain system as follows:

$$\partial u/\partial t = C_v\,(\partial^2 u/\partial z^2) + C_h\,(\partial^2 u/\partial r^2 + \partial u/r\partial r) + R(z,t) \qquad ...(2)$$

where C_v & C_h are the Coefficient of vertical and horizontal consolidation respectively and R(z,t) is the rate of loading.

The classical solution of Barron's Equation is based on the following assumptions:
(a) Each drain has a zone of influence represented by a circular cylindrical soil column of the same length as the drain and containing that volume of soil from which water can be assumed to be squeezed into the drain in question. The diameter 'D' of the dewatered cylinder is given by,
D = 1.05s for the drains placed in equilateral triangle grid &
D = 1.13s for the drains placed in square grid.
where 's' is the spacing between the drains.
(b) During consolidation, horizontal sections remains horizontal, i.e. following equal strain theory
(c) Permeability of the drain is infinite in comparison with that of the clay and
(d) Darcy's Law is valid.

From classical solution for a saturated soil we obtain

$$U_h = 1 - \exp(-8T_h/\mu) \qquad ...(3)$$

where, U_h = average degree of consolidation w.r.t. horizontal drainage,
Time factor, $T_h = C_h t/D^2$,

$$\text{so, } t = (D^2\mu/8C_h)\ \ln(1/1-U_h), \qquad ...(4)$$

$\mu \approx (n^2/1-n^2)\ [\ln(n)-0.75+1/n^2]$
$C_h = k_h/m_v\gamma_w$
$n=D/d$,
D = diameter of dewatered soil cylinder and
d = diameter of drain.
It has been shown by Hansbo that the Darcy's law is sometimes invalidated at small hydraulic gradients prevailing in practice in drained areas. With the help of permeability tests he showed that the relation between porewater flow 'v' and hydraulic gradient 'i' in this case followed that exponential law $v = Ki^n$ where K = coefficient of permeability in non-Darcian flow. He presented a new solution to the 'equal strain' consolidation theory based on this exponential law as he obtained the best agreement between the full-scale test results and this new theory for the exponent value n = 1.5. For this value the new theory gives,

$$t = (\alpha/\lambda)\ D^2\ (\sqrt{D\gamma_w/\Delta u_o})\{(1/\sqrt{(1-U_h)})-1\} \qquad ...(5)$$

where, Δu_o = average excess pore pressure at t=0
α = function of D/d
$\lambda = (K/m_v\gamma_w)$ = coefficient of consolidation in horizontal non-Darcian porewater flow. and can be taken approximately equal to C_v determined by Oedometer test.

In this equation the magnitude of consolidation load has an influence on the time of consolidation and the process of consolidation obtained from this equations is more rapid at the beginning, particularly on a site with soft highly plastic clay.

The radial consolidation equations include the drain diameter, d. A band-shaped PV drain must therefore be assigned an 'equivalent diameter', d_w. This is defined as the diameter of a circular drain which has the same theoretical radial drainage performance as the band-shaped drain. According to Kjellman, the draining effect of drain depends to a great extent upon the circumference of its cross-section, but very little upon its cross-sectional area. Following Hansbo, the equivalent diameter of a band-shaped drain with width 'a' and thickness 'b' can be expressed as, $d_w = 2(a+b)/\pi$. Subsequent FEM studies performed later by HALEY & ALDRICH (1986) suggests that it may be more appropriate to modify the above equation to $d_w = (a+b)/2$.

Well resistance of the drains is extremely large although the permeability coefficient of the drain material is much higher than the deposit. If it is assumed that the discharge capacity of the drain is q_w and that the permeability of the soil is k_h, Hansbo suggested that μ in the classical solution [eqn. 4] should be replaced by (neglecting last terms as value of 'n' is large in practice),

$$\mu \approx \ln(n) - 0.75 + \pi z\,(2l-z)\,k_h/q_w \qquad ...(6)$$

where, l = length of the drain when open at one end only (half length of the drain when open at both ends),
z = distance from open end of drain ($0 \le z \le 2l$) and
n as before.
It is quite obvious from the above equation that U_h varies with depth if there is drain resistance but is constant with depth if there is no well resistance.

The insertion of mandrel causes severe remoulding of the subsoil, mainly in the immediate vicinity of the mandrel but sometimes also at a fairly large distance from it. Evaluation of the disturbance effect is very complex. The present understanding is that disturbance, as it relates to drain performance, is most dependent upon: (a) Mandrel size and shape, (b) Soil macrofabric (soil layering) and (c) Installation procedure. For finding out the smear zone diameter, d_s, Hansbo suggested that for pre-fabricated drains, the cross-sectional area of the mandrel is replaced by an equally large circular area whose diameter is then doubled, that is, $d_s = 2d_m$, where d_m is equivalent diameter of mandrel. As remoulding leads to a decrease in the coefficient of consolidation and thereby to a delay in the consolidation process it has to be considered in the theoretical calculation. Hansbo (1979) suggested that μ in the eqn. (6) should be replaced by

$$\mu_s \approx \ln(n/s) + (k_h/k_h')\ \ln(s) - 0.75 + \pi z\,(2l-z)\,(k_h/q_w)\,[1 - \{(k_h/k_h').(n/s)^2\}] \qquad (7)$$

in which, $s = d_s/d$,
d_s = diameter of disturbed zone,
k_h' = permeability of the disturbed zone and other symbols are the same as before.

So, the general equation for the time 't' required to obtain a specified degree of consolidation 'U_h' is given by

$$t = (D^2/8C_h)\,(F(n) + F_s + F_r)\ \ln\{1/(1-U_h)\} \qquad ...(8)$$

where, $F(n) = \mu$ = drain spacing factor.
F_s is disturbance due to mandrel driving and
F_r is effect of well resistance.

For evaluation of pore pressure at a distance 'δ' from the center of the drain influence zone is given by Barron (1948) as:

$$u = u_{av}\,(\ln \delta/r_s - 2(\delta^2-r_s^2)/D_e^2 + (k_h/k_h')(n^2-s^2/n^2)\ln s)/\mu_s \quad ... (9)$$

where, $u_{av} = u_o \exp(-8T_h/\mu_s)$

Combined degree of consolidation for vertical and horizontal drainage can be taken into account by using the relation below (CARILLO, 1942):

$$U = U_v + U_h - U_vU_h \qquad ..(10)$$

Evaluation of Design Parameters

Application of the general equation requires the evaluation of soil properties C_h, k_h and k_h'. In general, it is considered appropriate to use soil property values evaluated at maximum vertical effective stress to be applied to the compressible soil in the field. Value of coefficient of horizontal consolidation (C_h) is generally higher than that of coefficient of vertical consolidation (C_v). In common practice, C_h has been estimated from C_v value measured in a standard Oedometer test. This is done by assuming an appropriate ratio of C_h/C_v. Generally this ratio may vary from 2 to 5. Hansbo (1979) suggested a value of around 2.5 to be used for the purpose. However, this ratio can be equal to unity or less in some places depending on higher water content of soil material.

Evaluation of the general equation requires an estimates of k_h/k_h'. This ratio is generally considered to range from 1 to 5 at strain levels anticipated within the disturbed soil. This ratio can be expected to vary with soil sensitivity and the presence or absence of soil macrofabric. Careful consideration, engineering judgement and possibly special testings are necessary to make realistic assessment of k_h/k_h' for a particular condition.

The necessity of evaluation of discharge capacity (q_w) of a PV drain is in the analysis of the drain resistance factor, which is almost always less significant than the drain spacing and disturbance factors. Accurate measurement of q_w is time consuming and requires relatively sophisticated laboratory testing. Vertical discharge capacities are often reported by drain manufacturers. Most of the cases the value of q_w is influenced by the confining pressure and changed shape of the drain under vertical compression.

GEOTECHNICAL INVESTIGATIONS

General

The whole area of Bangkok and its suburb is on the flat deltaic plain of Thailand where the subsoil is mainly soft marine clay. Inside the AIT campus, this soft clay deposites stretches upto about 8m depth which is underlain by the stiff clay layer and then the sand layer respectively. This layer is recent in age (about 2000 years old, COX, 1981). The top 2m of the soil has undergone weathering process and the natural ground water table is situated at a depth of about 1m (Fig. 1).

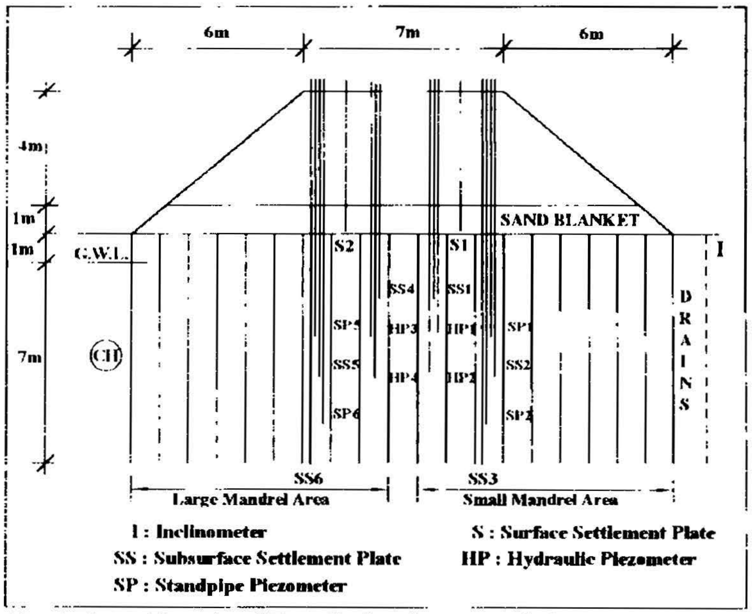

Fig. 1 Line Sketch of Test Embankment with Instrumentation

The sampling at the site has been divided into two categories. The first categories of samples were collected for the standard set of laboratory experiments with undisturbed soils and the other category of samples were used for the large scale consolidometer experiment for which disturbed sample of a large quantity (0.5m³) would do. The location for the undisturbed sampling is about 12m south of the test embankment. The samples were collected from the depths of 3m, 5m and 7m with using both large tube and standard Shelby tube. The second category of samples were collected from about 70m North-East of test embankment and from the depth of 2.3m. Back Hoe machine was used for the purpose.

Laboratory Tests

The subsoil between the depths of 3m and 7m is predominantly clay with high compressibility. The average natural moisture content of this soil is 75% with an average plasticity index of 50% with a decreasing trend towards the deeper zone. The average unit weight of the soil is 16 kN/m³. According to the Unified Classification System this soil falls under the class 'CH'. Compressibility characteristics are evaluated by using two kinds of tests viz., Oedometer test and Rowe Cell consolidation test. A loading period of 24 hours is selected. The results of both of the tests shows that the value of C_h is consistently higher than that of C_v. The average laboratory vane shear strength (S_{uv}) of the soil is found to be 3.5t/m². The CIU Triaxial test on undisturbed samples shows an average initial undrained modulus of 400t/m² which shows a close resemblance with empirical correlation '$E_u = 150\ S_{uv}$' suggested by previous researchers in the same area.

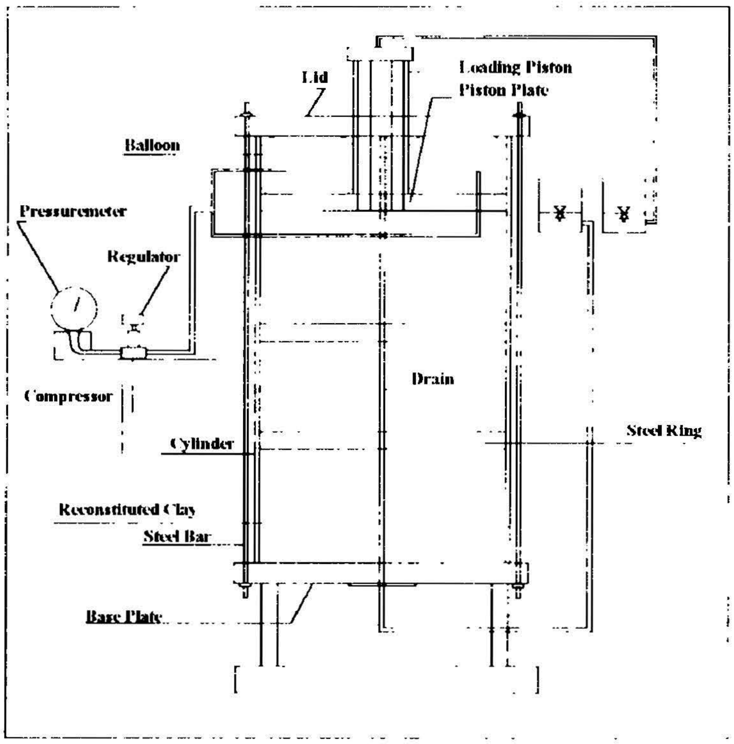

Fig. 2 Line Sketch of Large Scale Consolidometer

Large Scale Consolidation Test

Since in-situ structure or constitution of the sample is totally destroyed when we collect the disturbed sample from the field, it is always necessary to reconstitute this sample before the actual large scale consolidation test. The apparatus used for the test was designed in AIT, Bangkok and a line sketch of the apparatus is shown in Fig. 2. The main body of the apparatus consists of a large cylinder made of PVC transparent sheet of 1cm thickness with height 92cm and diameter 45.7cm. A PVC piston along with a middle shaft is generally placed over the top of the soil. The bottom of the instrument is equipped with a pulley to pull the mandrel down through the sample. Six balloons and the air compressor system was used for exerting the required pressure on the top of the piston. Teflon sheet was used in the inner surface of the cylinder to reduce friction between soil and cylinder. The process of reconstitution has been carried out in the large consolidometer itself in layers under a constant pressure of 10.2kN/m² for a period of 2 months. After that the small mandrel containing PV drain has been inserted for further consolidation test under a pressure of 47.8kN/m². Samples are collected and tested from both near the mandrel area and far from mandrel to study for the effect of smear due to insertion of mandrel.

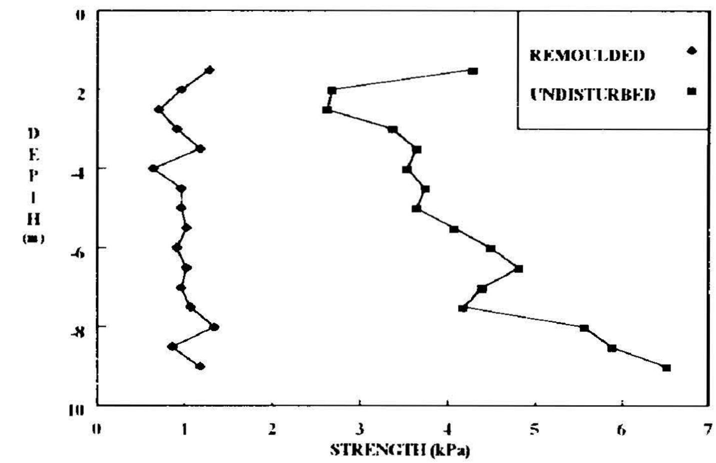

Fig. 3 Field Vane Shear Test Results

Field Tests

Sometimes it may happen that in some places the subsoil profile is such that it is very difficult to do the undisturbed sampling properly. Moreover, the time lag between the sampling and the laboratory testing can also put some inaccuracy into the test results. That is why it is always necessary to supplement the laboratory test results with the in-situ test results. Two different kind of tests were selected and performed in the field to show strength profile of the subsoil in-situ. Those are Field Vane Shear Test (Fig. 3) and Dutch Cone Penetration Test.

Field Monitoring of the Test Embankment

A test embankment was constructed in the AIT campus and was monitored to collect observed data from the field. The sketch of Fig. 1 is showing the test embankment along with the instrumentation. Mainly two types of data were collected regularly from this embankment viz, settlement and excess pore pressure to compare with the numerical model.

NUMERICAL MODELLING

A Finite Element numerical model (Fig. 4) of the transient, axisymmetric consolidation flow problem has been developed by considering equal strain and smear effect. The governing differential equation for vertical drain flow problem is shown in eqn. (2). The typical first order, time dependent differential equation was solved by discretising the time derivative by finite difference method which involves linear interpolation and fixed time steps Δt. The basic algorithm is written as,

$$KP[\theta u_1+(1-\theta)u_0] + PM[\theta(du_1/dt)+(1-\theta)(du_0/dt)] = 0 \quad ...(11)$$

leading to the following recurrence relation between timesteps '0' and '1':

$$(PM + \theta\Delta tKP)\, u_1 = [PM - (1-\theta)\Delta tKP]\, u_0 \quad ...(12)$$

Where 'KP' is the element stiffness matrix and 'PM' is the element mass matrix type. This system is only unconditionally 'stable' if $\theta \geq ½$. Common choice of the θ is 0.5 giving the 'Crank-Nicolson' scheme. sometimes $\theta = ½$ gives some initial fluctuation in result. It has been seen that a value of 0.6 could avoid this fluctuation with a little error incorporated with it. The different boundary conditions used in the model are as given below:

(i) The top layer is permeable, i.e. the excess pore pressure at any time $t>0$ is 0, i.e., $u_1 = 0$ at $z = 0$.
(ii) The bottom layer is impermeable, i.e., $\partial u/\partial z=0$ at $z=H$
(iii) The soil-drain interface boundary is permeable, i.e., $u = 0$ at $r = r_d$, where r_d is the drain radius and
(iv) No horizontal flow at the radius of influence, i.e., $\partial u/\partial r = 0$ at $r = r_e$, where r_e is the radius of influence zone.

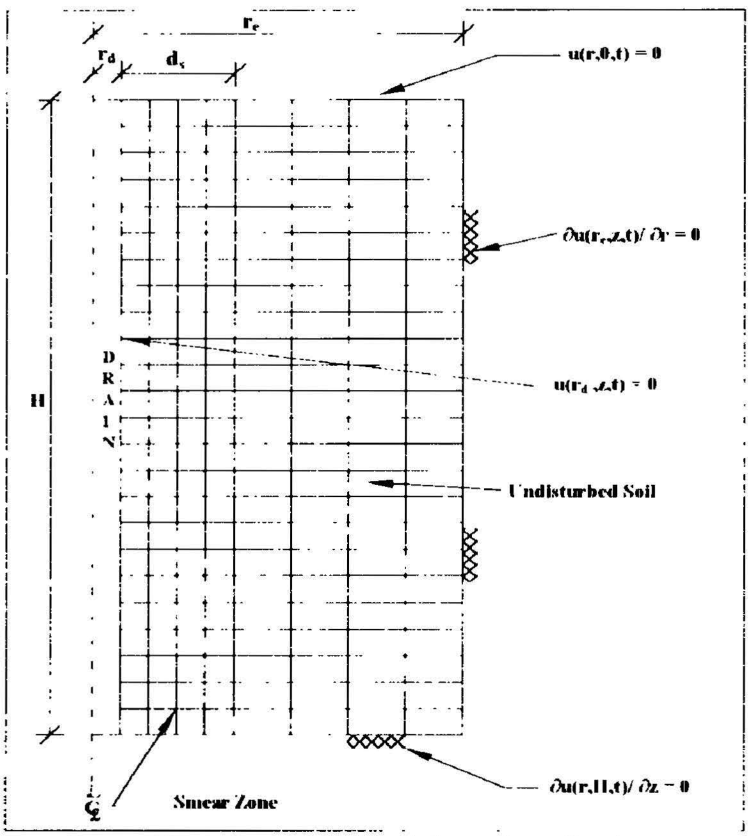

Fig. 4 Numerical Model with Boundary Conditions

The effect of smear has been treated in the model by assuming the smear zone to be also cylindrical about the drain with reduced permeability having a diameter of 4 to 6 times the equivalent radius of the driven mandrel. For this purpose the value of C_h/C_h' is necessary. In Finite element mesh, the smear zones are taken care of by changing sizes of the meshes to relatively finer in that zone and giving different soil property data for the elements in the zone.

Induced stresses due to superimposed load is calculated by using Gray's Equations and finite element package called 'ISBILD' using plastic theory. Initial excess pore pressure in each nodal point is then calculated by using either Henkel's equation or Skempton's equation. Degree of consolidation is calculated by averaging excess pore pressure in each element weighted by the area and is defined by the following equation:

$$U(t) = 1 - (u_0/u(t)) \quad ...(13)$$

where, u_0 = initial excess pore pressure and
$u(t)$ = excess pore pressure at any time instant $t>0$

The total settlement at any depth 'z', at any time 't' is calculated as follows,

$$S'_{tot}(t) = S'_{int} + S'_{con}(t) \quad ...(14)$$

Clearly which is nothing but the summation of initial and consolidation settlement at any time and depth. This program also generates excess nodal pore pressure at different depths in different time intervals as the output.

RESULTS AND DISCUSSION

Accuracy of prediction of performance of vertical drains by using numerical model is highly sensitive to the input parameters given to it. It has been seen that the larger the sample used higher is the value of consolidation coefficients found in the laboratory. For example, C_v or C_h found by Rowe cell consolidation is approximately 3 times that of Oedometer test. The analysis of the vertical drain time-dependent settlement using large scale consolidometer was done by using Hansbo's eqn. (8) considering smear effect only. The plot of Fig. 5 shows how it is compared with that of laboratory observations and numerical results. Hansbo's analysis considers the k_h/k_h' value as 1.5 and d_s=2d_m with a C_h value of 1.5m²/year found from laboratory. Hansbo's equation shows around 109 days time for 90% consolidation. However, since this method considers only horizontal drainage, it has overestimated the 90% consolidation time.

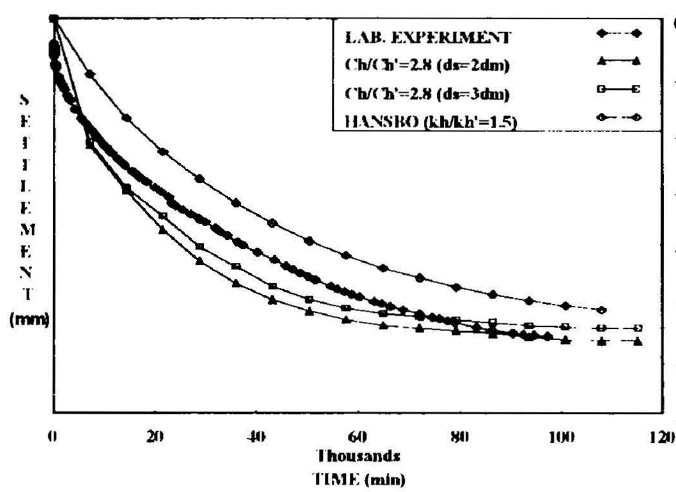

Fig. 5 Comparison of Observed & Predicted Settlement in Laboratory

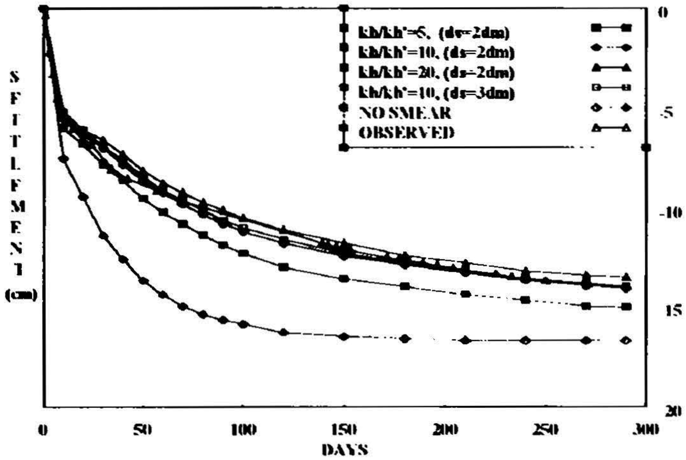

Fig. 6 Comparison of Observed & Predicted Settlement in Field in Large Mandrel Area at 5m Depth

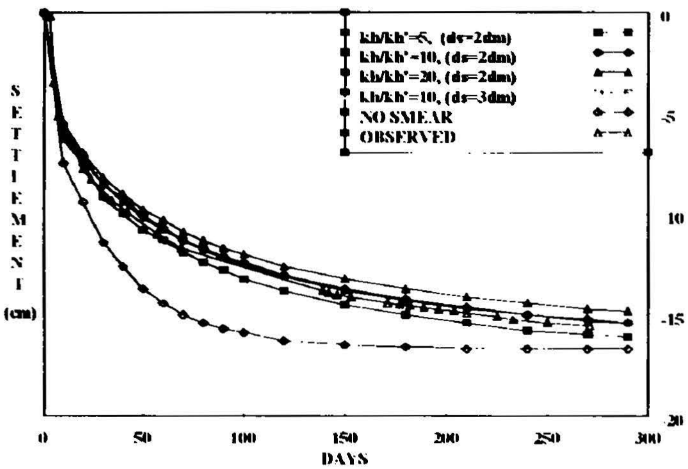

Fig. 7 Comparison of Observed & Predicted Settlement in Field in Small Mandrel Area at 5m Depth

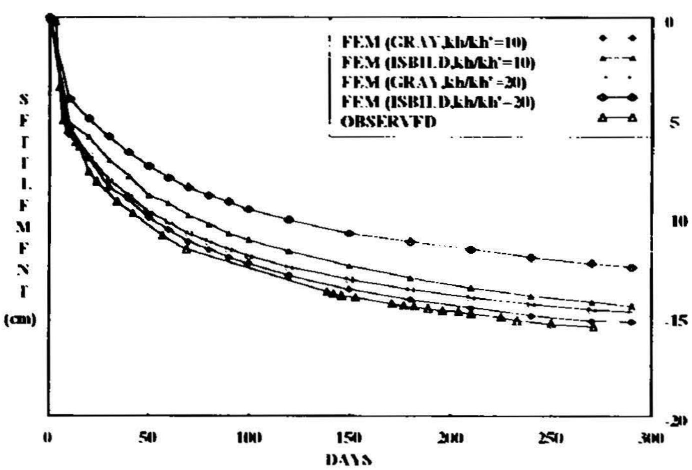

Fig. 8 Comparison of Settlement Results Using Gray's Eqns. & ISBILD Program in Small Mandrel Area at 5m Depth

The comparison plot of settlement calculated by the model and that observed in the field under large mandrel and small mandrel area is shown in the Fig. 6 and Fig. 7 respectively. It is clearly evident from those figures that the change in k_h/k_h' ratio affect the settlement rate mostly and larger the mandrel area more is this effect. The d_s/d_m ratio i.e. how much area is affected by the smear, influence least on the settlement rate. It can be seen that k_h/k_h' ratio generally falls in the range of 10.

The corresponding value of C_h/C_h' and C_h/C_v' were found from the laboratory curves to be 9 and 14 which were used as an input data to the model.

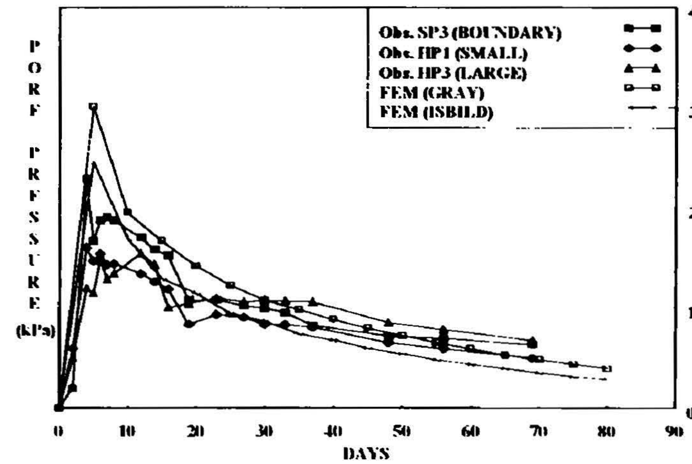

Fig. 9 Comparison of Observed & Predicted Pore Pressure in the Field at 3m Depth

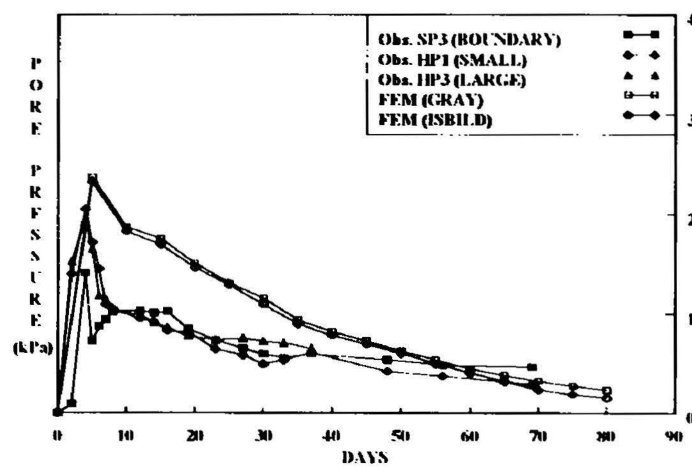

Fig. 10 Comparison of Observed & Predicted Pore Pressure in the Field at 6m Depth

Initial excess pore pressure calculated from the induced stresses given by ISBILD program gives a lower value than that by using Gray's Equations. From Fig. 8, it can be seen that the settlement results by using ISBILD and that by using Gray's Equations showed a similar trend. The difference shown by these two results in small mandrel area may be attributed to the lower initial excess pore pressure calculated by ISBILD and uncertainty in the input soil parameters.

To simulate the observed pore pressure with that calculated by the model (Fig. 9 & Fig. 10), we found some very high values of C_{field}/C_{lab} ratios particularly at the deeper zone, whereas, for the settlement prediction, relatively lower value seems to be reasonable. This can be mainly due to the unique location of the piezometer in the field which is at the middle of the four adjacent (1.2m spacing) drains and causes the low observed value. The other reasons may be presence of considerable amount of sand seams at after 5m depth, very high smear effect in the field and calculation of initial excess pore pressure by Skempton's method as in reality, stress condition under the embankment is not the triaxial because intermediate and minor principal stresses are not always equal. Another interesting feature is that the model shows a regular trend of pore pressure dissipation than the observed value. This is quite obvious since in the model we are using constant C_v and C_h values for a given depth, whereas, in nature this values are changing with time and consolidation. The pore pressure dissipation rate observed was quite regular in first three months but thereafter a sudden increase in pore pressure was observed. This might be due to the construction of another embankment in the vicinity and clogging of the piezometer. The calculated excess pore pressure contours for laboratory experiment and in the field are shown in Fig. 11 & Fig. 12 respectively. Figure 11 is self explanatory and the contours in Fig. 12 are drawn after 10 days, 1 month, 3 months and 7 months when the degree of consolidations are 59%, 68.6%, 82.8% and 92% respectively.

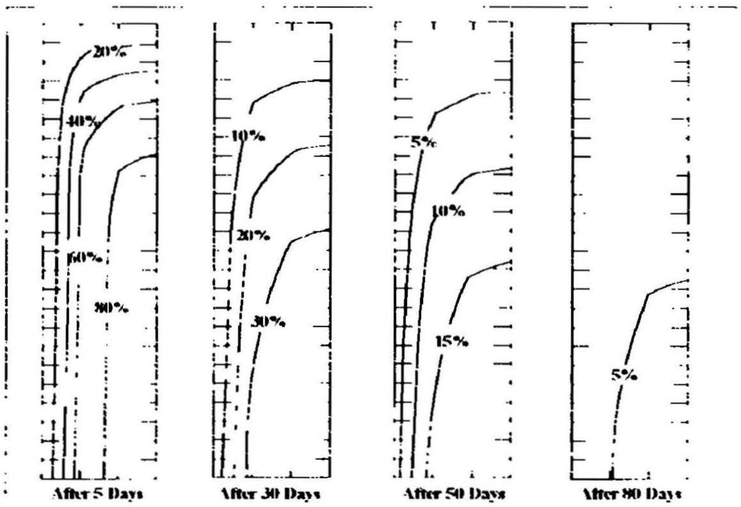

Fig. 11 Excess Pore Pressure Contours for Laboratory Expt.

CONCLUSIONS

A detailed review of all the results gives rise to one very important conclusion that the prediction of the equal strain finite element model is satisfactory for the behaviour of embankment over vertical drains. It is again proved the fact that the soil disturbance due to driven mandrel significantly affects the soil parameters of the surrounding subsoil and larger the size of mandrel more is this disturbance. For a given drain spacing and mandrel diameter, the effect of k_h/k_h' ratio on settlement is more prominent than the d_s/d_m ratio. A proposed value of k_h/k_h' around 10 with the combination of $d_s=2d_m$ has been found to predict the settlement much closer to the observed values in the field conditions The corresponding C_h/C_h' value found to be 9. For the best prediction of pore pressure in the field the ratio of C_{vfield}/C_{vlab} is found to vary from 15 to 60 with higher values towards the deeper zone. Since in the laboratory large scale consolidation, the soil sample has been reconstituted, prediction of settlement shows better conformity with excess pore pressure and degree of consolidation with $k_h/k_h'=2$ and $d_s=2d_m$.

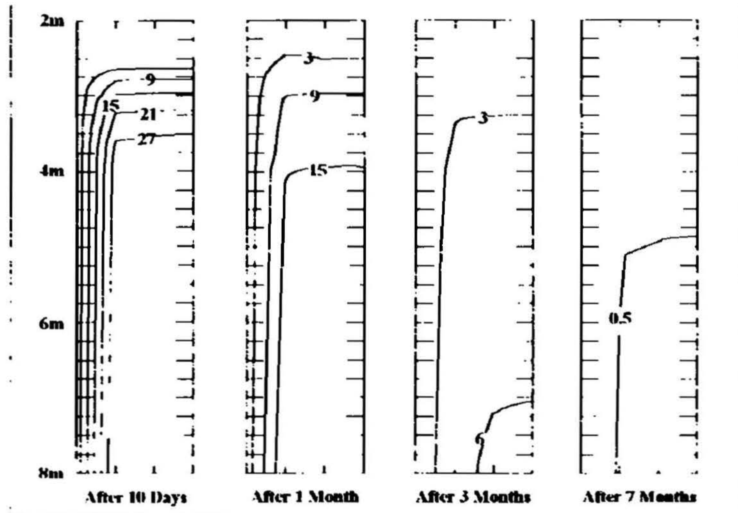

Fig. 12 Excess Pore Pressure Contours in the Field (kN/m²)

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